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Evaluation of reservoir operation strategies for irrigation in the Macul Basin, Ecuador

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1. Introduction

Irrigation projects are conceived for supplying water to agricultural lands where rainfall does not meet the crop water requirements during dry seasons or all year round (Withers and Vipond, 1980). The Macul Basin, located in the central part of the Ecuadorian coastal region (Fig. 1), consists of mainly agricultural lands and relatively small streams. The local economy is based on agriculture, livestock and fishing but these activities are at this moment rainfall dependent. Although the annual rainfall is 2000 mm, 80% of rainfall occurs between January and May. Currently, the crop production is limited to one harvest per year, which is also vulnerable to climate variability. In order to extend the growing season and to reduce the risk of crop failure, an irrigation project is planned for the Macul Basin.

Large irrigation projects aim at positive socio-economic impacts. This requires proper planning of reservoir operation strategies during the preconstruction stage. In deriving strategies an adequate relationship between water required for irrigation and for river environmental flows is attempted (Cai et al., 2003). Usually, reservoir operation strategies are tested by river/reservoir models as part of decision supporting tools for integrated water management (Loucks et al., 2005).

This research focuses on evaluating the reservoir operation management of the irrigation system as planned in the Macul Basin, Ecuador. The aim of our study it to design optimal reservoir operation strategies to reach a sustainable balance between irrigation demands, diverted flows, and respecting river ecological flows released from the reservoirs. For achieving this objective, a conceptual model is built for simulation of the integrated system, and applied in combination with a

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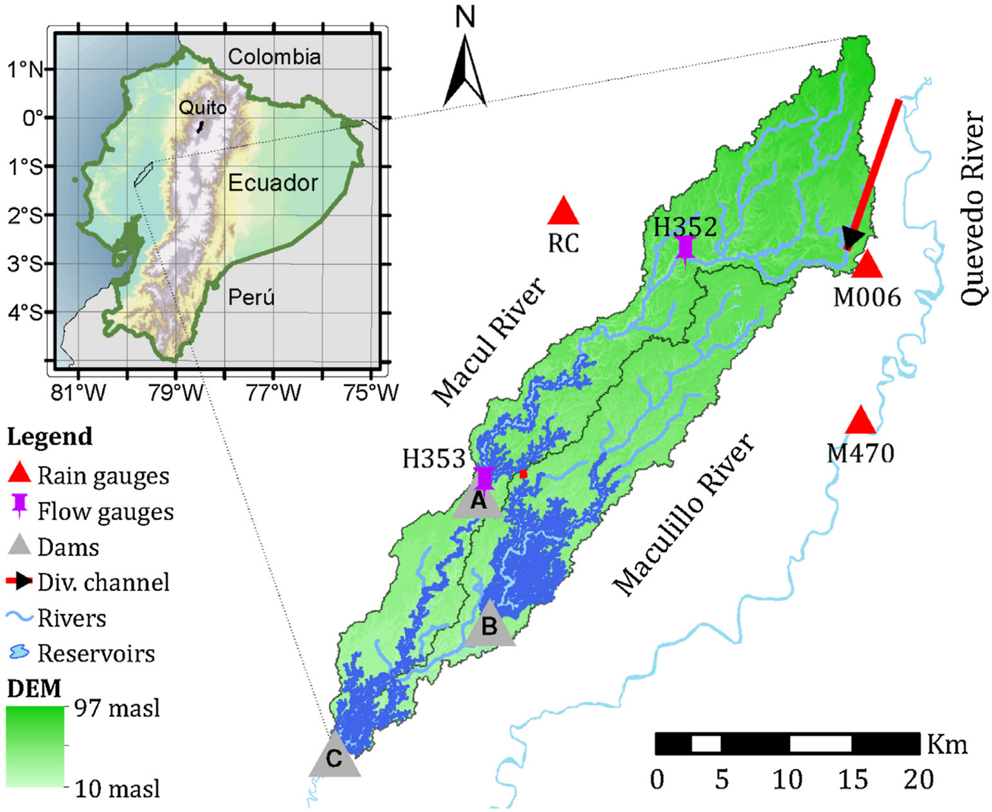


Fig. 1. Map of the Macul River Basin, Ecuador, showing its main components and the irrigation scheme. Reservoirs surface areas are presented at their maximum capacity. Arrows represent the diversion channels.

parameterised method for optimising the reservoirs' operation. In order to account for the long-term climate variability, long-term time series of hydro-meteorological data were used to force the model.

2. Case study and data

2.1. Case study description

The Macul Basin is located in Los Rios Province of Ecuador. It has an area of 604 km^2. Its geomorphology consists of medium hills and flat areas (PLR, 2013). The terrain elevation varies from 10 m to 100 m.a.s.l. The region has one of the warmest air temperatures in Ecuador ranging between 20 and 35 °C along the year. Water resources are abundant during the rainy season from January to May. The drainage system consists of two main perennial streams and several ephemeral tributaries. The main streams are the Macul and Maculillo Rivers, which during extreme rainy seasons have reached discharges up to 70 and 45 m^3/s, respectively; while during the dry seasons those rivers can have discharges below 1 m^3/s. There are approximately 45,000 inhabitants, among which 68% of the economically active persons work on agriculture, livestock or fishing (INEC, 2010; PLR, 2013).

The agricultural potential lands comprise fertile classes of Mollisol and Inceptisol soils, where multiple crops are sown. The crops are classified in short cycle and perennial crops. The short cycle crops are maize, soybean, rice, bean, groundnut, and watermelon. The perennial crops are cacao, banana, and palm. The integrated reservoir system planned in the Macul Basin aims to irrigate thirty thousand hectares of agricultural land. Its irrigation scheme aims distributing the water resources in time and in space along potential agricultural lands for extending the growing periods. For that purpose three detention dams (reservoirs) are planned: one at the upper Macul sub-basin, one at the Maculillo sub-basin, and the last one downstream the junction of the mentioned rivers (Fig. 1). Hereafter, those reservoirs are called A, B, and C respectively. Their features are summarised in Table 1. The total storage capacity of the system is $250 \times 106\,m^3$. PROMAS (2014) determined that the sub-basin contribution to the reservoirs is not enough to fill the reservoirs to maximum capacity during rainy seasons. Therefore, a diversion channel was designed to bring an additional average inflow of 10 m^3/s from the Quevedo River to the Macul River during the first five months of the year. The three reservoirs are connected either by natural streams or by an artificial channel (A–B), with the water flowing in the directions shown in Fig. 2.

Table 1
Reservoir system features.

Reservoir	A	B	C
Subbasin contribution area (km^2)	279.1	183.7	141.3
Dam height (m)	18	18	13
Reservoir maximum capacity (x10^6 m^3)	60.9	137	52.6
Irrigation area (ha)	7150	16242	6074

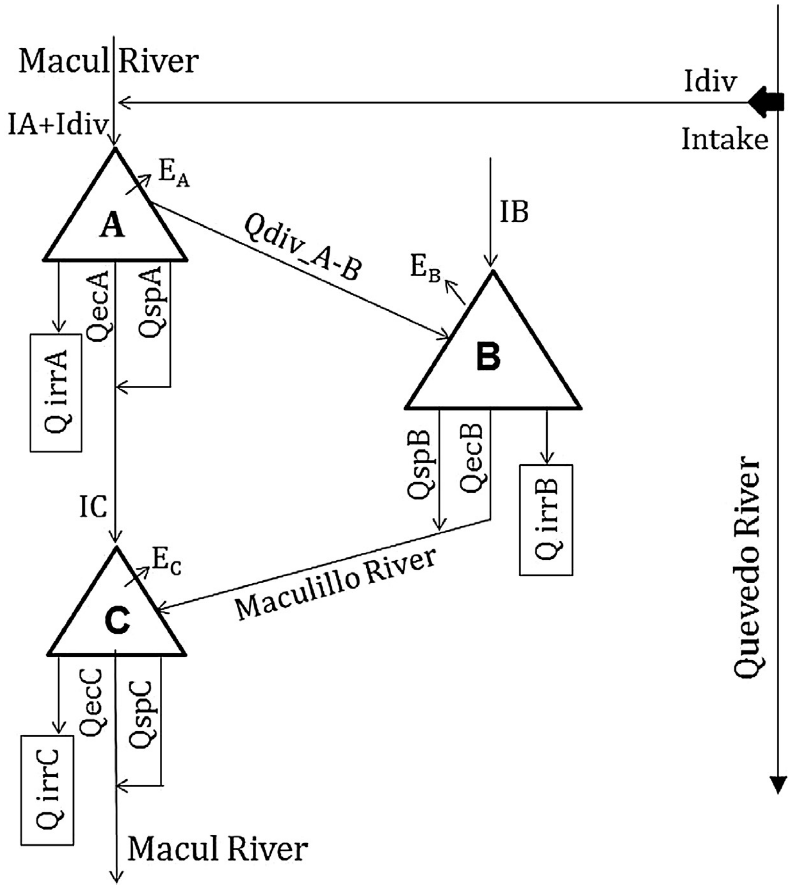


Fig. 2. Scheme of the reservoir system, showing inflows (*I*) and outflows (*Q*).

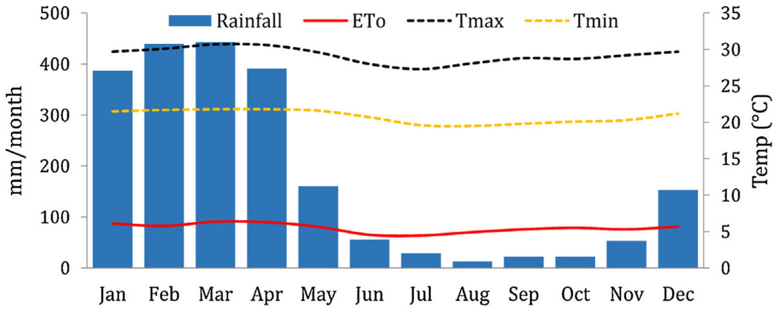
The planned irrigation system is by sprinkler with an application efficiency of 80%. The efficiency in the conveyance between the reservoirs and irrigated lands is 85% (PROMAS, 2014). Irrigation schedules were set for 16 h per day, and clustered in 10-day periods along the year. The demands increase between the years, because of the permanent crops.

2.2. Identification of variables

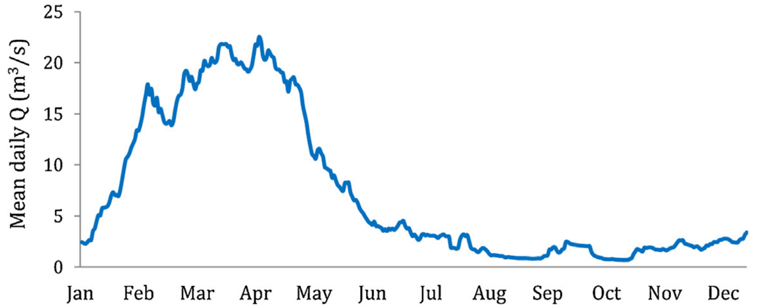
As a first step in building the reservoir operation model, the inflows and outflows of the system/reservoirs were identified, and long-term time series data searched for and/or considered for model simulation. The inflows consist of catchment runoff and diverted flows. The outflows consist of the irrigation demands, ecological river flows, spillway discharges, flows diverted from reservoir A to reservoir B, and reservoir evaporation. Fig. 2 gives a schematic representation of the system.

2.3. Geographical information

The geographical information such as for the rivers and channels, reservoirs, runoff subcatchments, rain gauges, flow gauges, soil distribution, land use, and irrigation areas were compiled in a GIS base file. Moreover, a digital elevation model (DEM) with a scale of 1:10,000 (IGM, 2011) and grid size of 5 m was used.



(a) Meteorological mean monthly time series data at station Pichilingue (M006)



(b) Mean daily flow from the Macul River observed at station Macul at Hda. Brasilia (H353)

Fig. 3. Hydro-meteorological mean monthly time series from selected stations in the Macul Basin. (a) Meteorological mean monthly time series data at station Pichilingue (M006). (b) Mean daily flow from the Macul River observed at station Macul at Hda. Brasilia (H353).

2.4. Hydro-meteorological information

The data described below were available on a daily time series basis. They belong to the "*Instituto Nacional de Meteorología e Hidrología*" (National Meteorology and Hydrology Institute—INAMHI, Ecuador). A period with historical measured data from 1965 to 2006 was selected. The time series contained some gaps and outliers. A careful data control was performed, interpreting and correcting some abnormal outliers.

a) River flow (Q): Two river flow gauging stations are available in the basin. They are located in the reservoir's A subbasin. Their names are Macul at Pte. Carretera (H352) and Macul at Hda. Brasilia (H353). The last one is located at the planned reservoir A.
b) Meteorological data: Air temperature, mean relative humidity, wind speed at 2 m height, and class A pan evaporation were collected at the Pichilingue (M006) meteorological station. Temperature data were complete, while the other mentioned variables presented several gaps. Reference potential evapotranspiration (ETo) was calculated with the FAO Penman–Monteith equation (Allen et al., 1998), after filling up the gaps.
c) Rainfall (P): Two rain gauges at Pichilingue (M006) and Rio Congo cover the Macul subbasin. They received weights according to the Thiessen polygon method.

Gaps of missing data were filled in by rainfall generation, and catchment runoff modelling (for river flows at the gauging station), applying the lumped conceptual catchment runoff model described in Section 3.6. Charts with the hydro-meteorological mean monthly time series data are shown in Fig. 3.

3. Methods

3.1. River/reservoir simulation and optimisation modelling

River/reservoir models can be classified in two groups: simulation and optimisation models, although many optimisation models include simulation (Wurbs, 2005). Both practices share the objective of being decision supporting tools for developing or evaluating reservoir operation rules for single or multiple purpose water management systems. Those models aim for optimum water allocation to users and minimising risks such as water shortages, floods or environmental impacts. The central equation is the water balance. The simulation models are used to compare reservoir performance by testing of alternative scenarios, mainly alternative operations of the reservoir (i.e. calibrating valve/gate operations). Optimisation

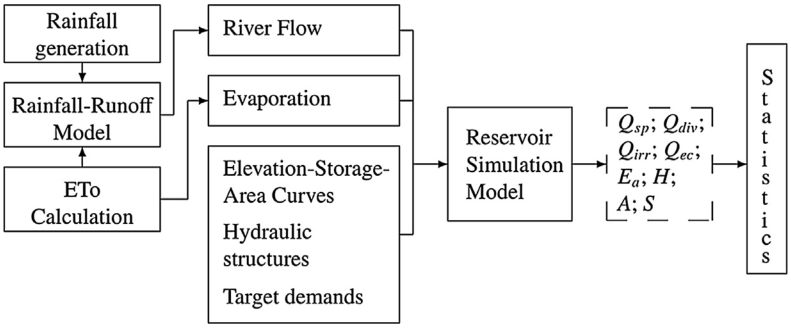


Fig. 4. Components in the conceptual model per reservoir in the system. The reservoir simulation model calculates the variables shown in the dashed box based on the inputs shown in the second column. In case that the sub-basin is ungauged or the observed data have missing gaps, the data is generated by the auxiliary models shown in the first column of the scheme. Finally, statistical analysis is performed to the reservoir simulation results for decision making support.

models seek automatically for the optimal performance by a predefined computational algorithm (Wurbs, 1993) (i.e. artificial neural network).

Advances in reservoir modelling mainly focused on flood control, water supply to households and hydropower. Fewer research studies have been done for irrigation water supply, although some representative studies in this field exist (e.g. Al-Ansari et al., 2013; Kumar et al., 2006; Amir and Fisher, 1999; Afshar et al., 1991). A possible reason is that the agricultural sector is more flexible than other water management sectors. Water shortages in agriculture have a less drastic impact as compared to shortages on drinking water or on hydropower (Amir and Fisher, 1999). The fact that the economical returns are lower in irrigation supply might be another reason.

An important gap identified from the literature review is that existing approaches and studies most often do not account for the long term climate variability. This variability can be considered by simulating long-term time series of hydro-meteorological data. It allows to statistically analyse the response of the system to any of the hydro-meteorological events in the time series. In this study, historical meteorological time series were available for the period 1965–2006. Simulation of such long series can only be performed in a practical way when parsimonious conceptual models are applied for the water system components. They describe the system response behaviour to the system inputs in a simplified, strongly macroscopic way by conceptualising and lumping the most dominant system processes. They can be identified and calibrated based on simulation results obtained by historical time series data.

3.2. Conceptual water system model

Fig. 4 shows the components of the conceptual model per reservoir in the system. The central component is the conceptual reservoir simulation model that routes the inflows (I) through the reservoir to obtain reservoir storage volume, water level, and outflow (Q) results (Chow et al., 1988). The central equation is the continuity equation, Eq. (1), which is expressed as the changes in the water level H versus time t, depending on the reservoir area A:

$$\frac{dH}{dt} = \frac{I(t) - Q(t,H)}{\mathrm{A}(H)} = f(t,H) \tag{1}$$

Fenton (1992) and Fiorentini and Orlandini (2013) have reviewed/analysed different existent methods for solving the nonlinear first-order ordinary differential Eq. (1). Fenton (1992) concluded that the traditional level pool routing method fails to be an accurate method. He compared the Runge–Kutta (RK) methods for solving Eq. (1) and found a good accuracy in the second order RK results as compared to higher orders. In addition, the second order method is stable and robust. This method was therefore implemented in this study.

In Eqs. (1) and (2), and Fig. 4, Q is the total reservoir outflow discharge, which comprises the environmental/ecological discharge Q_{ec}, irrigation demand Q_{irr}, and spillway discharge Q_{sp}. Reservoir evaporation E also was added to the total reservoir outflow. E is calculated in function of the surface reservoir area. In addition, Reservoir A has an extra discharge, which is the flow diverted to reservoir B: $Q_{\mathrm{divA\_B}}$. Consequently, Q is expressed as:

$$Q = Q_{\mathrm{sp}} + Q_{\mathrm{divA_B}} + Q_{\mathrm{irr}} + Q_{\mathrm{ec}} + E \tag{2}$$

The reservoir outflow discharges depend on the reservoir water level H, for which five zones are defined (based on the reservoir outflow structures), as Fig. 5 illustrates. There are three fixed dam levels and one variable threshold which define the zones. The fixed dam levels are: (i) the spillway crest H_{crest}, (ii) the diversion channel bottom H_{bdiv}, and (iii) the reservoir bottom outlet H_{bo}. The variable threshold is named $H_{\mathrm{irr/ec}}$, which defines the irrigation storage volume below which irrigation is restricted in order to minimise the ecological impacts downstream the dams. The zone located below H_{bo} is the inactive zone, which is assigned for sediments accumulation S_{sed}.

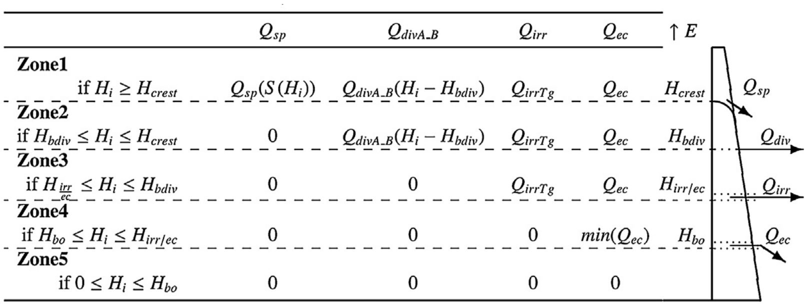


Fig. 5. Operational rules/water allocation of reservoir A. Reservoirs B and C present the same scheme, with the difference that they do not have zone 2. It is included in zone 3 in those reservoirs.

Table 2
Calibration parameters and constrains of the reservoir model.

Symbol	Description	Units
Q_{maxA_B}	Max. Q diverted from reservoir A–B if $H_A \geq H_{crest}$	m^3/s
Q_{divA_B}	Max. Q diverted from reservoir A–B if $H_{crest} \geq H_A \geq H_{bdiv}$	m^3/s
a_A	Constant to define Sec_A by Eq. (3)	–
a_B	Constant to define Sec_B by Eq. (3)	–
a_C	Constant to define Sec_C by Eq. (3)	–

Zone 4 for the reservoir water level, which is bound between $H_{irr/ec}$ and H_{bo} (Fig. 5), reserves a security ecological volume, S_{ec}. This zone restricts the reservoir water use and releases Q_{ec} until the reservoir water level recuperates the $H_{irr/ec}$ level. The storage capacity of this zone, S_{ec}, is calculated as:

$$S_{ec} = \min(Q_{ec}) \times a \times dt + S_{sed} \tag{3}$$

The time step dt in the present study equals 1 day. A dimensionless parameter a is created for optimising the reservoir operation. The term $a \times dt$ represents the period of time during which extreme low inflows occur in the subbasin with magnitudes proportional to the minimum river ecological flows $\min(Q_{ec})$. A threshold of ten percent of the mean monthly river flow was considered to be the lowest limit for the ecological flow. The threshold $H_{irr/ec}$ is obtained in function of S_{ec} by applying the elevation—storage relation for each reservoir.

3.3. System operation description

As shown in Fig. 2, the flow distribution in the system works as follows: the inflow to reservoir A (IA) is composed of the sub catchment A runoff (I_{Ao}) plus the flow diverted into the system I_{div}. The outflow discharges from that reservoir are given in Eq. (2). The discharge released from reservoir A to the river is $Q_{Ariver} = Q_{Asp} + Q_{Aec}$. Applying the same nomenclature, the inflow to reservoir B is $I_B = I_{Bo} + Q_{divA\_B}$. Finally, the contribution to reservoir C is composed of its own catchment contribution plus the releases from reservoirs A and B: $I_C = I_{Co} + Q_{Ariver} + Q_{Briver}$, which is equal to $I_C = I_{Co} + Q_{Asp} + Q_{Aec} + Q_{Bsp} + Q_{Bec}$.

The flows diverted from reservoir A to reservoir B need to be controlled and optimised for filling the three reservoirs during the rainy seasons. Reservoir A receives the flow diverted from Quevedo River (I_{div}) and is the first, most upstream one that distributes the water to the other reservoirs. This is performed with two constrains: Q_{maxA_B} and Q_{divA_B}, which determine the limits for diverting the water to reservoir B, depending on whether the reservoir level is above or below the spillway crest level H_{crest}, as shown in Table 2. The flow excesses of reservoirs A and B fill reservoir C.

The routing times of the flows between the reservoirs as well as the routing times along the diversion channel are assigned after analysis of the response of these system components. Hydrodynamic routing by solving the full hydrodynamic equations, i.e. Saint–Venant equations, was not implemented because it would strongly increase the computational times, hence would avoid long-term simulations to be conducted in a reasonable time. Moreover, the model has a daily timestep, so that the hydrodynamic aspect becomes less critical. Therefore, a conceptual, reservoir-type based routing method was applied. The flow routing time from A to C, and from B to C through the rivers Macul and Maculillo were determined based on the peak flow time shift between the stations H352 and H353 along the Macul River. The flows diverted from A to B were determined by the water surface profiles of the gradually varied flow through the diversion channel. Their routing times were assumed less than one time step, or in other words the flow that enters the channel at A reaches B in the same time step. This assumption was based on the total travel distance of 1.5 km, the mild slope, and the channel bed material being concrete.

3.4. *Reservoir model optimisation*

The goals of the reservoir optimisation are: (i) to find an optimal operation for the diversion of flows and for filling the three reservoirs during the rainy season, and (ii) to find the optimal $H_{irr/ec}$ thresholds for each reservoir in order to reach an optimal balance between the irrigation demand (or other water use) and ecological sustainability. The optimisation consists in a trial-and-error process with the following steps:

1. River/reservoir system modelling for a period of historical hydro–meteorological conditions.
2. Statistical post-processing of resultant reservoir outflows and water levels.
3. Optimisation of the values for the parameters and constrains listed in Table 2. The maximum diversion flows Q_{maxA_B} and Q_{divA_B} are constrains which define the diverted flows from A to B. The parameters a determine the $H_{irr/ec}$ thresholds for the three reservoirs, hence the allocation of water for irrigation and river environmental flows releases.
   a) Optimisation of the water distribution constrains, Q_{divA_B} and Q_{maxA_B}, is performed by analysing the resultant time series, ensuring that the reservoirs most often reach full capacity. The important simulated variables for analysing the water distribution are: H, Q_{sp}, Q_{div}, and Q_{irr}.
   b) Simultaneously with the analysis done in 3.a for filling the reservoirs, the parameters a are selected with the goal to maximise the irrigation during the dry season and preserving the river environmental flow releases. This trial-and-error process is done by trying parameter a values and analysing the annual irrigation volumes. Regarding the river environmental release, this is performed by analysing the annual minimum simulated reservoir water levels. An extreme value analysis of those reservoir levels is performed (see Section 3.5) for determining a convenient/required return period for a dry spill, which happens when the reservoir level drops below the level H_{bo}.

3.5. *Statistical analysis*

The reservoir simulation model results are statistically analysed and compared with the target values from the case study after aggregation of the results at annual, monthly and daily time scales. Since the most critical operation conditions in an irrigation project occur during droughts, extreme value analyses were performed on the annual minimum simulated reservoir water levels. The exponential extreme value distribution was calibrated to these annual minima after transforming the water levels H to their inverse values ($1/H$), according to Taye and Willems (2011).

3.6. *Rainfall-runoff modelling*

A parsimonious lumped conceptual rainfall-runoff model was identified and calibrated in a step-wise, data-based way, following the VHM approach by Willems (2014). The model was applied for filling in the gaps in the river flow measurement at station H353.

The rainfall-runoff model describes the catchment rainfall—runoff with four lumped processes, which are overland flow, subsurface flow, base flow and catchment storage. The splitting of the total catchment runoff is based on the historical

Table 3
Reservoir operation settings and parameters for selected simulations, together with some results that are important for deciding on the operation rules.

	Parameters					Max. storage[a]			Mean annual date of Smax[b]			Virr actual/Virr tg[c]			Days Qeco > Qecmin[d]		
No	Q_{divA_B} (m^3/s)	Q_{maxA_B} (m^3/s)	a_A	a_B	a_C	A (%)	B (%)	C (%)	A	B	C	A (%)	B (%)	C (%)	A (%)	B (%)	C (%)
1	16	16	0	0	0	100	98.05	83.37	April/8	May/30	April/30	77.86	85.84	84.64	86.78	85.95	99.60
2	10	16	0	0	0	100	96.67	84.03	March/14	May/30	May/1	81.82	85.23	84.88	87.97	85.63	99.62
3	5	16	0	0	0	100	97.80	84.99	March/4	May/27	April/30	86.42	86.79	86.36	89.41	86.26	99.62
4	5	10	0	0	0	100	92.99	87.45	March/4	May/30	April/30	86.35	86.05	87.24	89.12	87.32	99.69
5	5	10	15	0	0	100	92.53	87.43	March/4	May/30	April/30	85.33	82.99	87.24	99.91	84.75	99.60
6	5	16	15	0	0	100	98.14	84.98	March/4	May/27	April/30	85.33	86.60	86.39	98.68	86.02	99.61
7	5	16	30	0	0	100	97.90	85.00	March/4	May/30	April/30	85.04	85.77	86.41	99.95	85.81	99.55
8	5	16	60	0	0	100	97.93	84.99	March/4	May/26	April/30	84.70	87.09	86.45	100.0	86.26	99.60
9	5	16	15	15	0	100	98.16	85.00	March/4	May/28	April/30	85.33	85.03	86.51	99.93	95.65	99.68
10	5	16	15	30	0	100	98.17	85.00	March/4	May/28	April/30	85.33	84.29	86.51	99.93	99.02	99.69
11	5	16	15	60	0	100	98.17	85.02	March/4	May/28	April/30	85.33	84.16	86.57	99.93	99.81	99.70
12	5	16	15	120	0	100	98.19	85.04	March/4	May/30	April/30	85.33	84.01	86.57	99.93	99.93	99.69
13	5	16	15	60	60	100	98.17	85.32	March/4	May/28	April/30	85.33	84.16	85.03	99.93	99.81	99.99
14	5	10	15	60	15	100	92.61	87.50	March/4	May/30	April/30	85.33	80.48	87.06	99.91	99.74	99.99
15	5	16	15	60	30	100	98.17	85.16	March/4	May/28	April/30	85.33	84.16	85.78	99.93	99.81	99.99
16	5	16	15	60	15	100	98.17	85.09	March/4	May/28	April/30	85.33	84.16	86.14	99.93	99.81	99.99

[a] Ratio between mean yearly maximum reservoir storage in the 42 years of simulation and the reservoirs maximum capacity.
[b] Mean annual date that each reservoir reaches its maximum storage.
[c] Ratio between the mean annual actual irrigation volume and the irrigation target volume.
[d] Percentage of days in the 42 years of simulation that the ecological discharge released form the reservoir is above the minimum river ecological discharge.

observed flows at the gauging station. The lumped soil water capacity is assessed by cumulating the catchment rainfall minus the total runoff flows and evapotranspiration losses. The rainfall amount that contributes to each of the subflows depends on the soil water content for each time step; also these dependencies are identified in a data-based way. See Willems (2014) for details on the method.

The runoff to the reservoirs B and C was generated proportional to the runoff calibrated at the gauging station by the contribution areas.

4. Results

4.1. Operational rules diversion channel

After analysing the long-term simulation results for the period 1965–2006 (daily time scale), first the diverted discharges Q_{divA_B} and Q_{maxA_B} were determined for filling the reservoir during the rainy season. There is enough head for diverting a flow equal to the maximum diversion channel capacity (16 m^3/s) when reservoir A is at full capacity. However, it was found that setting restrictions was better to reach the goal of filling the three reservoirs during rainy seasons.

When the reservoir A level is below the spillway crest a maximum diverted discharge of 5 m^3/s (Q_{divA_B}) is allowed. While, when the reservoir A level is above the spillway crest, the operation is controlled to 16 m^3/s (Q_{maxA_B}). Those rules help to fill reservoir A earlier, and then to distribute the water to the other reservoirs more effectively. If the Q_{divA_B} control value would not be set, high flows would be continuously diverted from reservoir A to reservoir B. It would cause a delay in the filling of reservoir A, and would not use the reservoir C capacity that effectively, as the flow will take longer to arrive to that reservoir (see simulation 1 in Table 3).

Some of the selected simulations are shown in Table 3. It demonstrates the model's ability to act as a decision support system. More specifically, the simulations shown in the Table demonstrate how the good reservoir operations and parameter choices were identified in a step-wise way: mean yearly maximum reservoir storage, mean annual date that each reservoir reaches the maximum storage, ratio between the actual irrigation volume and the target irrigation volume, and the number of days that the ecological discharge released from the reservoirs is above the minimum ecological river flow. A good balance between maximizing the irrigation and conserving the river ecological flow was found in the simulation No. 16.

Mean daily simulated reservoir water levels and their standard deviations are shown in Section 4.2.1. Section 4.2.2 presents the annual maximum irrigation volume available with three different parameter a_A values. The flow composition from reservoir A is given in Section 4.2.3.

4.2. River/reservoir system simulation results

4.2.1. Water levels

Fig. 6 shows the mean and standard deviation of the daily water levels in reservoirs A, B and C. Reservoir A reaches its full storage capacity every year in March, while there is a high variability of the water levels during the dry season. November is the most critical month for the irrigation water availability.

Reservoir C shows similar behaviour as reservoir A. It reaches its full storage capacity in the first week of April, but with a higher standard deviation than reservoir A. Reservoir B has some years were it does not reach its full storage capacity (typically between April and the second week of May).

For the optimised parameter a values, the return period with which the annual minimum water level reaches the bottom outlet equals 80 years for reservoir A (Figs. 7 and 8), 10 years for reservoir B, and 120 years for reservoir C.

The $H_{irr/ec}$ thresholds ensure releasing the minimum environmental flow during dry seasons in between the mentioned returns periods. Even though the reservoir water levels drop the $H_{irr/ec}$ thresholds almost every simulation year in all reservoirs, it happens for short periods of time along the year, usually during the critical month of November. Water level duration curves show that the selected thresholds for each reservoir exceed around 87% of the days in the 42 years simulation for reservoir A (Fig. 7).

4.2.2. Evaluation of the project's target irrigation demand

The available water for a sustainable irrigation for a minimum negative ecological impact downstream by respecting the ecological flow, is presented in Figs. 9 and 10. The simulation results show a drop in actual irrigation with reference to the target irrigation demand starting at the end of August. After optimisation of the parameter values a for the three reservoirs, with the goal to maximise the irrigation, the results show that the planned irrigation demand is high for all the three reservoirs. The optimised values of the parameters a for the three reservoirs are $a_A = 15$, $a_B = 60$ and $a_C = 15$. The annual target irrigation volumes simulated for these parameter values for the period 1965–2006 are 72.68, 178.43 and 66.72×10^6 m^3 in reservoirs A, B, and C, respectively. Those are 14.7%, 15.8% and 13.9% higher than the resulting available volumes for irrigation.

4.2.3. Flow routing and flow composition

Fig. 11 shows the mean daily simulated flows routed through the reservoir A and operated with the parameter $a_A = 15$, and with the constraints given in Section 4.1. Fig. 11 also shows each of components of the reservoir in- and outflow discharges.

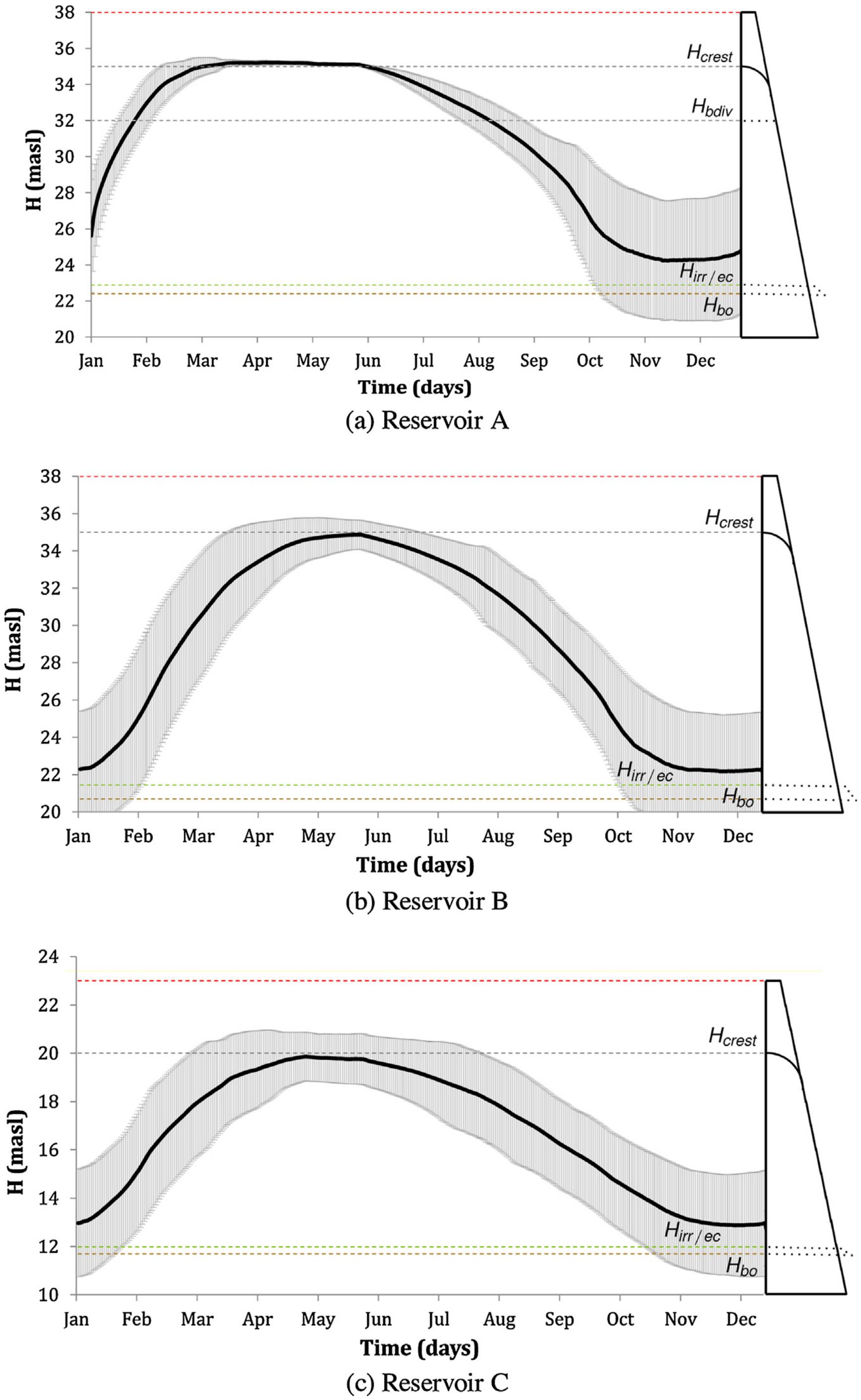


Fig. 6. Mean and standard deviation of simulated daily water levels in reservoirs A, B and C. (a) Reservoirs A. (b) Reservoirs B. (c) Reservoirs C.

The spillway discharges are high during the rainy season (January to May). Diversion flows occurs until the reservoir level H_A drops the bottom level H_{bdiv} of the diversion channel A_B. That happens most often in July. When the spillway discharges Q_{Asp} are high, the diversion discharges reach the constrain $Q_{maxA\_B} = 16\,m^3/s$. Irrigation flows obviously are high during the dry season and depending on the stored volume. The ecological river discharge is variable from month to month. The reservoir evaporation is found to be significant; it reaches an equivalent value of $1\,m^3/s$ when the reservoir is at maximum capacity.

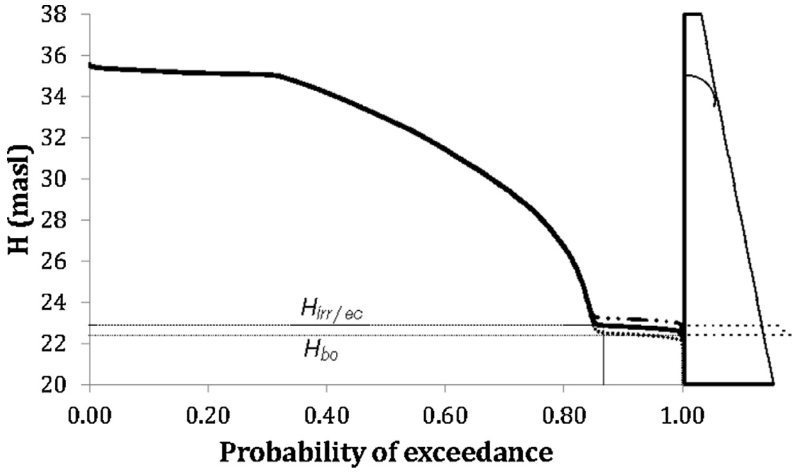


Fig. 7. Water level duration curve for reservoir A. $H_{irr/ec}$ is exceeded 87% of the days during the 42 simulated years.

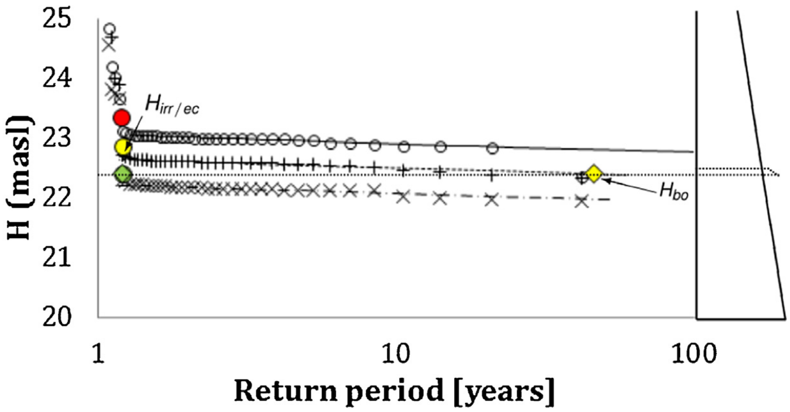


Fig. 8. Return period of the annual minimum water levels in reservoir A, after three different parameter *a* values. The optimal thresholds $H_{irr/ec}$ and H_{bo} for $a = 15$ are highlighted.

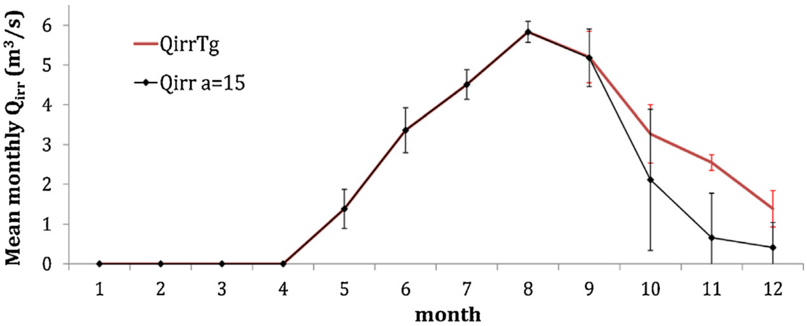


Fig. 9. Mean and standard deviation of monthly target and simulated irrigation volumes from the reservoir A, after application of parameter $a = 15$.

5. Discussion

5.1. Analysis of the river/reservoir simulation results

The optimum diversion constrains allow continuous water releases for filling reservoir B without affecting the other flows released from reservoir A while the water level is below the spillway crest. Furthermore, when the water level in A is above the spillway crest, diverting higher discharges helps to optimise the use of the available water in the system instead of evacuating all the excess water by the spillway to C.

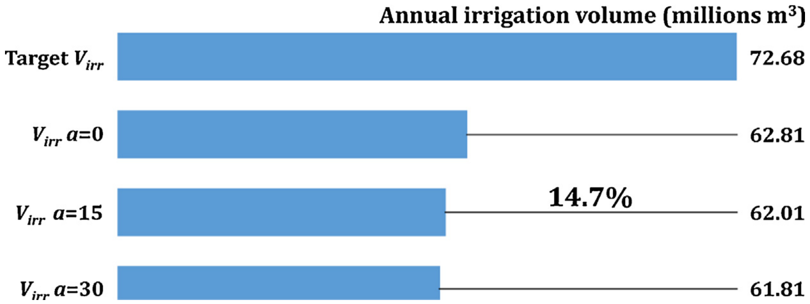


Fig. 10. Annual target and simulated irrigation volume from reservoir A, after application of three different parameter *a* values.

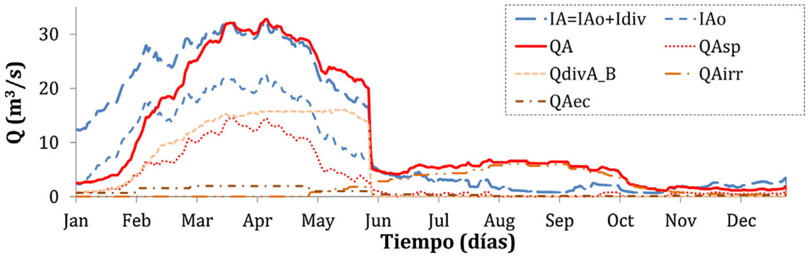


Fig. 11. Composition of the mean daily flows routed through reservoir A averaged for the period 1965–2006.

No set of parameter *a* values could be obtained that allows to meet the actual irrigation demand (Fig. 9). A drop in the actual irrigation demand is found, which usually starts around September, while the most critical month for irrigating is November.

The selection of the better parameter values and reservoir operation strategy strongly depends on the reference terms considered in the study. The annual irrigation volumes are found to be very sensitive to the selected $H_{\mathrm{irr/ec}}$ thresholds tested with different parameter *a* values. However, the differences between the resultant irrigation volumes (Fig. 10) are considerable smaller than the difference with the irrigation target volumes. The optimal irrigation volume for a sustainable irrigation is only 15.2% of the target irrigation volume of the entire reservoir system. This draws to the conclusion that the planned irrigation is too high or the designed system of reservoirs insufficient. The annual water available for irrigation found is 62.01, 150.17 and 57.47 10^6 m^3 for reservoirs A, B, and C respectively.

In order to find possible solutions for limiting the deficit in irrigation volume, an obvious solution is shrinking the irrigated land. This is not a desired solution, because it might have negative socio-economic impacts on the project beneficiaries. Two other actions are proposed here instead:

(i) The deficit in irrigation can be solved by increasing elevation of the spillway crest by 0.86 m, 1.09 m, and 0.57 m in the reservoirs A, B, and C respectively. It is equivalent to an increase of 704 ha of the total water area covered by the reservoirs. It would be a good option to be implemented because it does not imply significant changes to the original project proposed. The feasibility of the given suggestion would depend on the project budget. In addition, it also has to be analysed if it is feasible to increase the reservoir area without negative effects to the land use in the surroundings of the reservoir.

(ii) A second solution is to implement deficit irrigation schedules. This does not implicate changes in the designed reservoirs. It would lead to increased water productivity by using less water and still producing relatively high and stable crop yields. Simulation of the effects of such deficit irrigation would require comparison of the yield crops obtained with the original planned irrigation demand against the ones obtained with deficit irrigation. Those analyses might be aided by crop models such as AquaCrop (Geerts et al., 2010). The planned deficit irrigation schedules would need to be evaluated by the reservoir system model and the reservoir operation rules adjusted correspondingly.

5.2. Model strengths and threats/limitations

The approach for reservoir optimisation followed in this study by parameterising the system with one parameter per reservoir and two per diverted channel is a simple and practical one. The current reservoir model fixes the storage zones for the reservoir operation purposes, in this case just irrigation, by defining a zone that stores a volume of water to guarantee minimal river environmental flow in extreme dry conditions.

The proposed approach requires the model to be combined with long-term simulations and extreme value analysis on the extreme flow conditions. The optimisation was in the current study based on a trial-and-error process leaving the final decision to the user, but based on prior defined objectives. The approach can be easily extended for use in larger or multi-purpose reservoir applications. Other, more automated approaches would be the application of genetic algorithms (Lerma et al., 2013), or Monte Carlo Analysis (Rossi et al., 2011).

6. Concluding remarks and recommendations

Results show that the irrigation volume demand as planned for the overall system exceeded the available actual annual volume of water for having sustainable irrigation by 15.2%. The current research proposed sustainable reservoir operation strategies and quantified the available water volumes for irrigation in each reservoir. Even after optimisation, the project goals of supplying irrigation water to all the planned beneficiaries could not be met. Shrinking the irrigated land is not a desired solution, because it might have negative socio-economic impacts on the project beneficiaries. Possible solutions are (i) changes in the spillway height for increasing the reservoirs storage capacity, which is a feasible solution because it does not involve significant changes to a project that is still in the design stage; (ii) installation of a deficit irrigation schedule, which most likely is a successful solution for the large community of irrigators who will benefit from the project (22,512 families).

The constructed reservoir model can be generalised to any single purpose reservoir system. In addition, the auxiliary models applied allow to handle gaps of missing hydro-meteorological data inputs. Therefore, this set of integrated modelling components is considered very useful as supporting tool for integrated water reservoir project planning. The approach is applicable also to other and more complex systems in order to optimise irrigation works.

Conflict of interest

None.

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Appendix A. Supplementary data

Supplementary data associated with this article can be found, in the online version, at http://dx.doi.org/10.1016/j.ejrh.2015.12.063.

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